RATIONAL DESIGN AND CONSTRUCTION OF CORRUGATED STEEL WEB PC BRIDGE HAVING SUBSTRUCTURE WITH IMPROVED SECTIONAL FORCE

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ABSTRACT

Nishita Bridge is a PC box-girder bridge with a corrugated steel web, and it was built on Ban-etsu Expressway as part of the project to widen the expressway to four lanes. In the construction of this bridge, in order to improve the ground reaction and pier sectional force attributable to the statically indeterminate force of the superstructure, double-wall-type bridge piers were adopted and the horizontal reaction were adjusted. This is the nation's first corrugated steel web PC bridge for which double-wall-type piers are adopted and in which the horizontal reaction is adjusted. Also, the construction of the overhang was rationalized to achieve a significant reduction of the work period.

This report presents rational design and construction implemented for Nishita Bridge.

1. INTRODUCTION

Nishita Bridge is a four-span continuous PC rigid frame box-girder bridge with a corrugated steel web, and it was built on Ban-etsu Expressway as part of the project to widen the expressway to four lanes. Because of factors that restricted the erection method, such as the road geometric alignment and the national highway and Japan Railway's trains under the elevated road, the structure type of the second stage line bridge was changed from the steel arch bridge used for the first stage line bridge to a continuous PC rigid frame corrugated steel web with an overhang. It was expected that there would be the following problems in the structure of the bridge: (1) since the ratio of the pier height to the fixed span length would be small in the structure of the rigid frame, statically indeterminate force of the superstructure would put a large strain on the piers; (2) since the existing arch abutments commonly used for both the first and second stage line bridges (photos 1 and 2) would be used for the foundation of the rigid frame pier for cost reduction, the ground reaction needed to be improved. Therefore, improvement of the sectional force in the substructure was sought through the adoption of double-wall-type rigid frame piers (photo 3) and the application of the horizontal reaction to the

superstructure. This is the nation's first corrugated steel PC web bridge for which double-wall-type rigid frame bridge piers were adopted and to which the horizontal reaction was applied.

In addition, since the bridge would be build over Route 288 and Japan Railway's Ban-etsu East Line, the safety management needed to be taken into account during the erection of the main girder for the transportation under the elevated road, and thus, there was need for a reduction of the work period. Therefore, to achieve a significant reduction of the work period, facilitation of construction was sought through the application of load to the corrugated steel member during construction.

Photo 1. Existing arch abutment (pier P2)

Photo 2. Existing arch abutment (pier P3)

Photo 3. Double-wall-type piers

2. WORK SUMMARY

(1) Location

As shown in Figure 1, Nishita Bridge is a bridge built on the expressway that crosses the Tohoku district in the mainland from east to west.

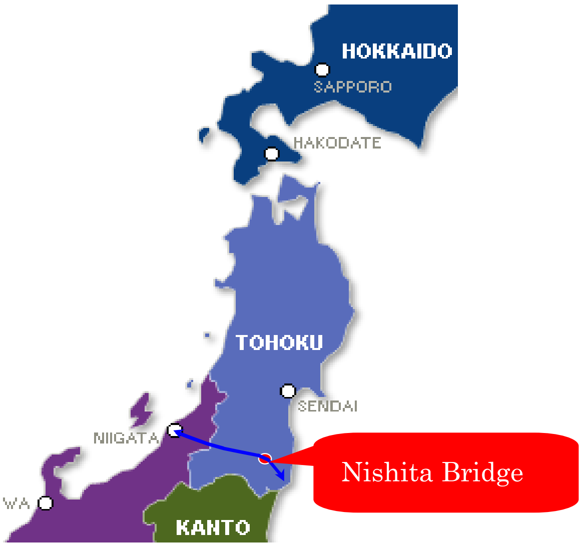


Figure 1. Location of Nishita Bridge

(2) Overview

The work summary of the bridge is as follows. Figs. 2 and 3 show a general view of the bridge and a sectional view of the main girder, respectively. Fig. 4 shows construction steps of the bridge.

Location: Nishita-machi, Koriyama, Fukushima

Superstructure: Four-span continuous PC rigid frame box-girder bridge with corrugated steel web

Substructure: Abutments A1 and A2, reversed T-type abutment (cast-in-place concrete shaft)
Pier P1, RC column-type pier (cast-in-place concrete shaft)
Piers P2 and P3, RC double-wall-type rigid frame pier
(spread foundation) *Foundation of each of the piers P2 and P3 is used for both upbound and downbound lines.

Total bridge length: 263.2 m

Span length: 34.4 + 66.75 + 114.5 + 45.15 m

Effective width: 8.750 m

Horizontal alignment: R = 1000 m

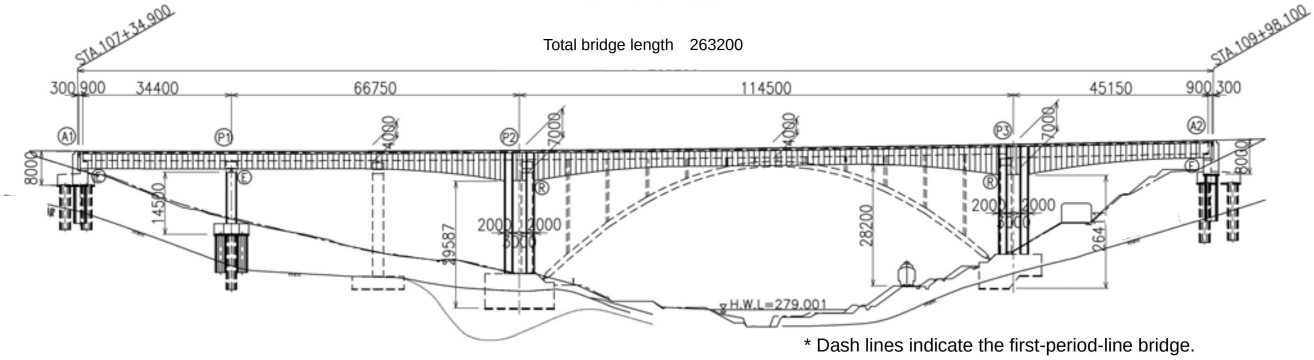


Figure 2. General view of the bridge

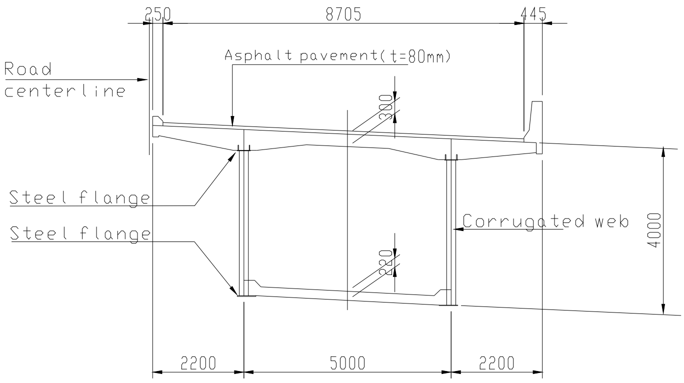


Figure 3. Sectional view of the main girder

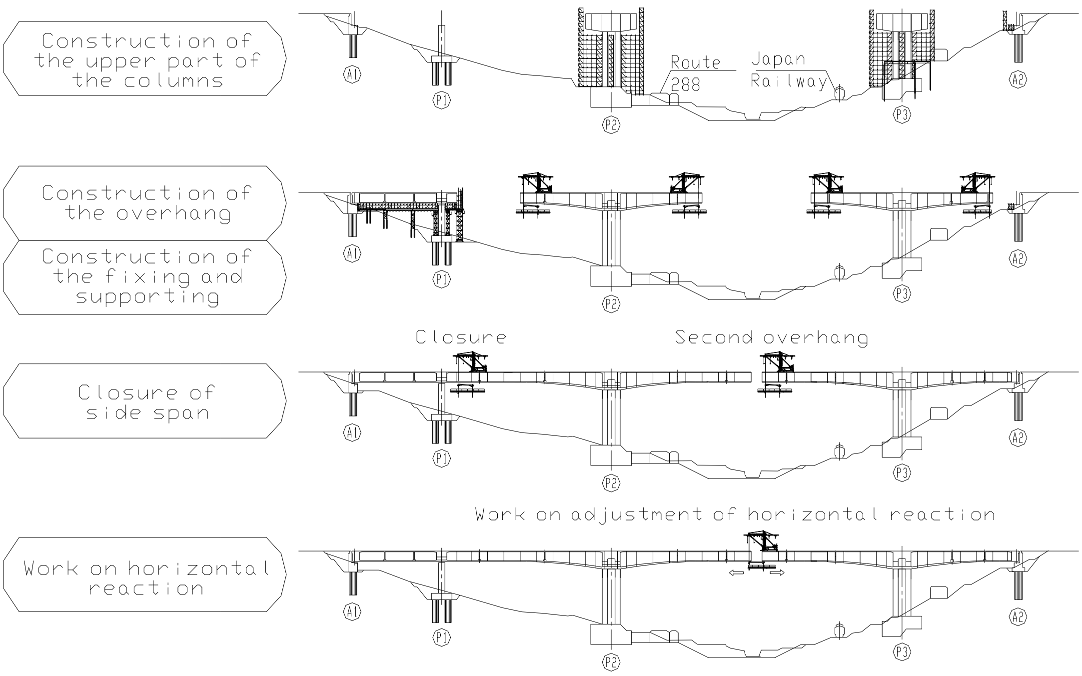


Figure 4. Construction steps

3. PROBLEMS AND SOLUTIONS CONCERNING THE CONSTRUCTION

Problems in constructing the bridge and solutions to the problems are as follows:

(1) Problems

(a) Improvement of the ground reaction was needed to use existing arch abutments as the foundation of the continuous rigid frame bridge.

(b) Improvement of the piers' sectional force generated by creep of the superstructure or drying shrinkage was needed because of a relatively small height of the pier height with respect to the span length.

(c) To construct the bridge crossing Route 288 and Japan Railway's Ban-etsu East Line, a reduction of the work period was needed for the safety management.

(2) Solutions

(a) Double-wall-type rigid frame piers were adopted for the piers P2 and P3 to reduce the sectional force.

(b) Prior to the closure of the center between the piers P2 and P3, horizontal reaction was applied to the superstructure to adjust the reaction force.

(c) Load was applied to the corrugated steel member during construction to facilitate the construction of the overhang.

4. SOLUTIONS TO STRUCTURAL PROBLEMS

4.1. Double-wall-type rigid frame pier

(1) Overview

As to the double-wall-type rigid frame pier, as compared with a conventional single-column rigid frame pier, by reducing the flexural rigidity of the pier along the bridge axis, it is possible to reduce the sectional force, which is caused by such factors as earthquakes, creep of the superstructure, drying shrinkage, or temperature change and which affects the pier. Further, as the flexural rigidity of the rigid frame pier decreases, restraining force on the deformation caused by expansion and contraction of the superstructure also decreases, and thus the sectional force that affects the superstructure can be reduced.

(2) Effects provided by the improvement of the sectional force of the two-wall-type piers

Deformation of the double-wall-type rigid frame pier caused by the horizontal reaction differs from that of conventional piers, and Fig. 5 shows an example of such deformation. When a single-column pier is deformed, the upper part of the pier is caused to be slanted by the horizontal reaction. In contrast, in the case of the two-wall-type pier, the upper part of the pier will not be slanted

by the horizontal reaction but deformed in a horizontal direction. Thus, while the superstructure of the single-column rigid frame pier bends because of the rotational deformation at the corner of the rigid frame pier affected by the horizontal reaction, since the double-wall-type rigid frame pier deforms horizontally, the superstructure of the pier is more resistant to such deformation. When horizontal reaction is applied to the two-wall-type pier, the pier reacts to the horizontal reaction mainly by changing the axial force. Between the single-column rigid frame pier and the double-wall-type rigid frame pier, the sectional force at the corner that intersects with the main girder and the sectional force at the base of the pier in the case of an earthquake were compared, and the Table 1 shows the comparison results. It is found that adoption of the double-wall-type rigid frame pier significantly reduces the sectional force of both the superstructure and the substructure.

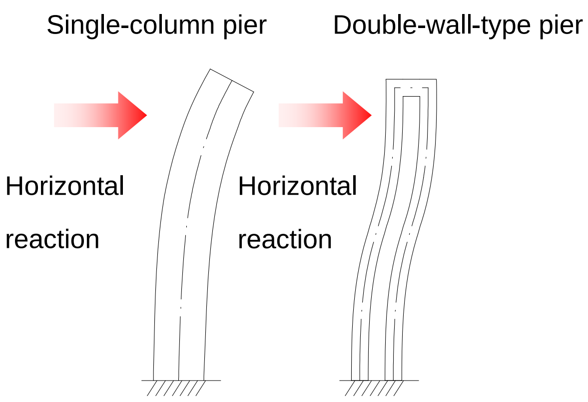


Figure 5. Deformation of piers

Table 1. Comparison of sectional force at level -1 earthquake (single load)

	Main girder's rigid frame corner (Bending checking on cross section)		Pier		
	Single-column pier	Double-wall-type pier	Single-column pier	Double-wall-type pier: Left wall	Double-wall-type pier: Right wall
M(kNm)	-19200	-8500	109500	38900	39100
4N(kN)	4900	5200	400	-12300	12100

4.2. ADJUSTMENT OF HORIZONTAL REACTION

(1) Overview

In this construction, load that negates the deformation or the sectional force generated in the substructure by the statically indeterminate force of the completed superstructure was given during construction, so as to improve the deformation or the sectional force of the substructure when the bridge is constructed.

In this construction, as shown in Fig. 6, before closing the center, horizontal reaction was applied to the cross section of the main girder to improve the cross section of the substructure.

Analysis conducted in advance indicated that the application of horizontal reaction may cause cracks to the top and bottom ends of the piers P2 and P3. For this reason, to cope with a decrease in rigidity attributable to cracks, there was an option to use a nonlinear analysis in design. However, conventional linear analysis was used. This is because the nonlinear analysis for designing used at combination level requires quantitative assessment of variation in characteristics of concrete

material, and safety design results cannot always be obtained. However, to manage displacement in actual construction, it is necessary to grasp deformation by taking into account a reduction in rigidity generated by cracks in the pier. Thus, a nonlinear analysis was conducted with a focus on when horizontal reaction was applied.

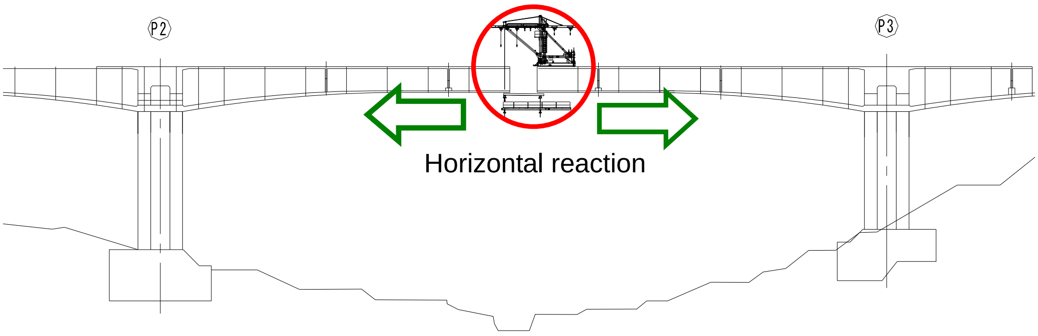
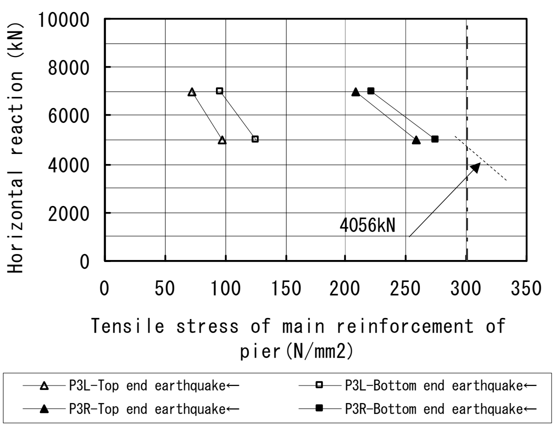


Figure 6. Horizontal reaction

(2) Applied load and load structure

1) Applied load

The applied load for the adjustment of horizontal reaction was determined based on; the relationship (1) between the horizontal reaction and the ground reaction of the footing; and the relationship (2) between the horizontal reaction and the stress of the axial reinforcement of the pier.



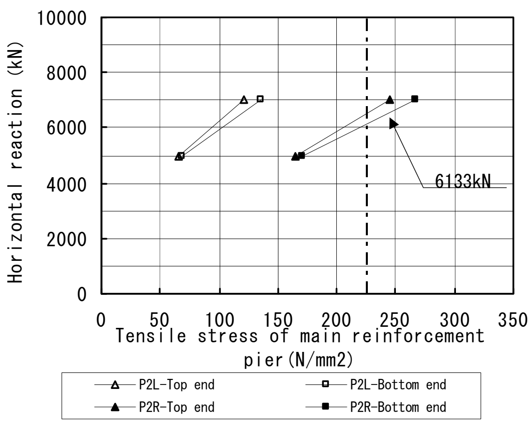


Fig. 7 Horizontal reaction and stress (when completed)

Fig. 8 Horizontal reaction and stress (during construction)

As a result of examination, checking (2) was found to be critical. Figs. 7 and 8 are relationship diagrams of checking (2). The application of horizontal reaction was managed in the range of 4100 to 6000 kN.

2) Load structure

Regarding the loading method, in order to reduce additional bending of the top and bottom floor slabs or corrugated steel caused by the application of load, the top and bottom floor slabs were provided with protrusions, and load was applied to four points per cross section. As to the loading structure, as shown in Fig. 9 and photo 4, a total of 8 (including 4 spares) 3000-kN hydraulic jacks were arranged, and each of the top and bottom floor slabs was handled by 2 jacks controlled by a single hydraulic pump, so that load was independently applied to the top and bottom floor slabs.

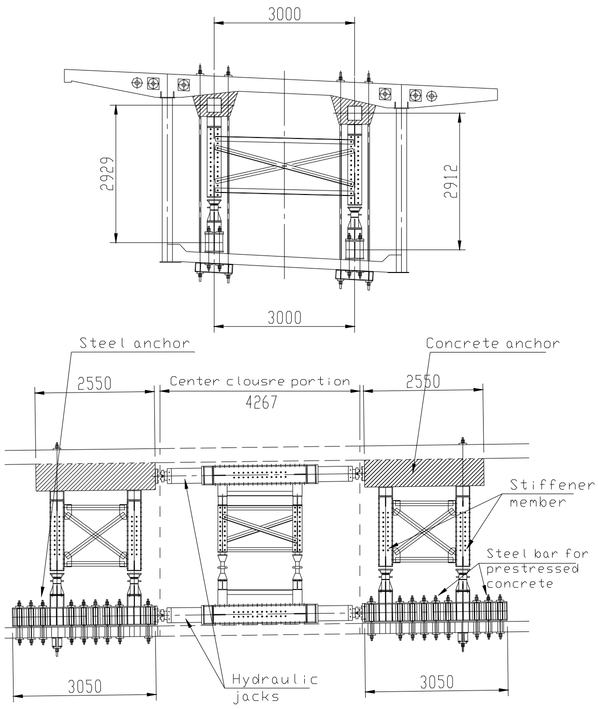


Figure 9. Application of horizontal reaction

Photo 4. Horizontal reaction loading device

4.3. CONSTRUCTION MANAGEMENT METHOD

(1) Measurement

Regarding the displacement caused when horizontal reaction is loaded, displacement along the bridge axis and displacement in the vertical direction at each support portion, upper part of the columns P2 and P3, and cross sections to which horizontal reaction was applied were measured by a displacement meter. Regarding the measurement of displacement in the vertical direction, displacement of the loaded cross section where deformation is distinct was measured with a level. A reinforcement strain gauge was previously set to the axial reinforcement of the pier during the construction of the pier, to measure strain of the reinforcement.

(2) Construction management method

Since the purpose of applying horizontal reaction was to improve the sectional force of the

substructure, the management method was based on load management.

4.4. MEASUREMENT RESULTS

(1) Load- displacement

Fig. 10 shows the relationship between the horizontal reaction and the horizontal displacement of the cross section on the P3 bridge pier side to which the horizontal reaction was applied. Measured displacement values were somewhat smaller than analytic values. It is assumed that this is the difference between the actual strength and the design value, since the analytical elasticity modulus of the pier was set based on design strength. In the nonlinear analysis, bending moment that causes cracks was reached at 2500 kN. In actual construction, visible cracks were generated when 3000 kN was applied, which was approximately consistent with the analytic value. As to the maximum load, while no problematic points were seen in terms of the amount of displacement, crack width, and stress intensity of reinforcement, since the work progress of the floor slabs reached control limits, loading was ended at 5500 kN.

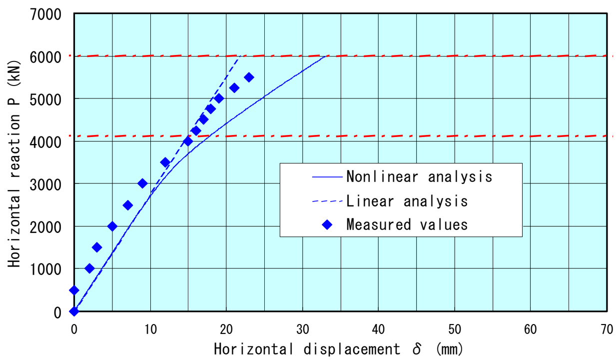


Figure 10. Horizontal reaction and displacement

(2) Stress intensity of reinforcement

Fig. 11 shows the results of comparison between the design values and the measured values of the stress intensity of the axial reinforcement of the upper part of the pier P3. The measured values of the tensile stress of the reinforcement were approximately below 50% of the design values. It is conceivable that this is because, in designing RC pier through the linear analysis in which the entire cross section is effective, (1) rigidity decreased by cracks is ignored and (2) the tensile strength of concrete is ignored.

Additionally, according to the design value history diagram shown in Fig. 11, it is assumed that the stress intensity of reinforcement of the pier decreases along with the progress of the creep of the superstructure and drying shrinkage. This is due to the previously applied horizontal reaction, and thus, improvement of sectional force through the adjustment of horizontal reaction was confirmed.

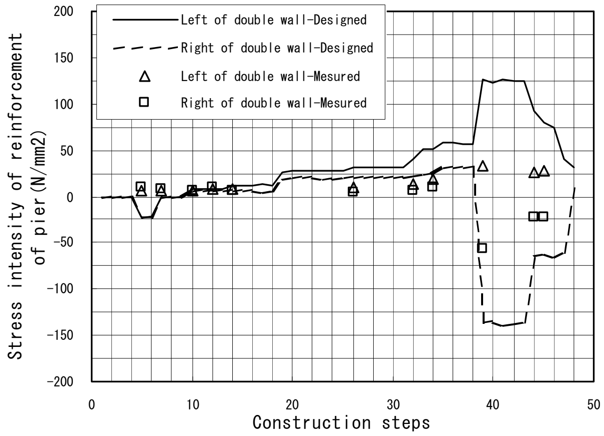


Figure 11. History diagram of stress of axial reinforcement of pier

(3) Crack width

Cracks were generated at all the bases of the top and bottom ends of the piers P2 and P3. The maximum value of the crack width when load was applied was 0.3 mm, and the crack width was reduced to 0.25 mm after tension of external cables. While it was expected that the crack width would decrease to such an extent that did not affect durability along with the progress of the creep of the superstructure and drying shrinkage, waterproof property was considered, and preventive maintenance measures were taken by using permeable waterproof material.

5. SOLUTION TO THE PROBLEM CONCERNING SAFETY MANAGEMENT

5.1 Construction of overhang

Conventionally, when constructing an overhang, the entire concrete placing load is supported by a movable construction vehicle. However, in the construction of the overhang of the bridge, as shown in Fig. 12, since corrugated steel top and bottom flanges were formed continuously with splice plates, the concrete placing load of the top floor slab was supported by the mobile construction vehicle and the concrete placing load of the bottom floor slab was supported by the corrugated steel. As shown in Fig. 13, a beam member was set on the truss member by using the mobile construction vehicle. Also, as to the method for supporting the concrete form for the bottom floor slab, a load support beam was disposed at the top end of the corrugated steel (see photo 5), and the form for the bottom floor slab was hung from the support beam. Thus, without any large-scale modification of the mobile construction vehicle, the block length was increased from the conventional 4.8 m to 6.4 m, which contributed to the reduction of the number of blocks. Thus, about 40 days of the work period in the construction of the overhang was shortened.

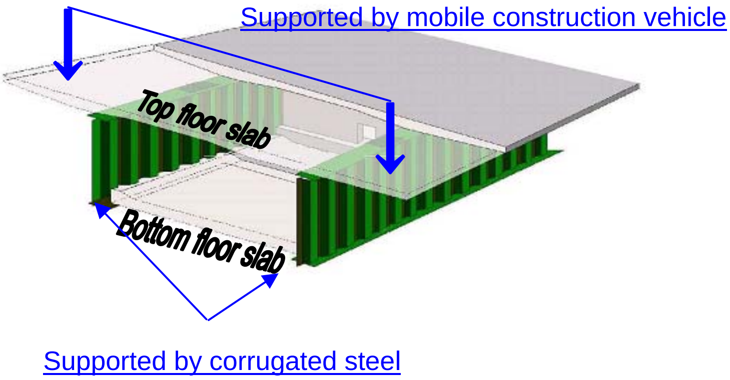


Figure 12. Construction of overhang

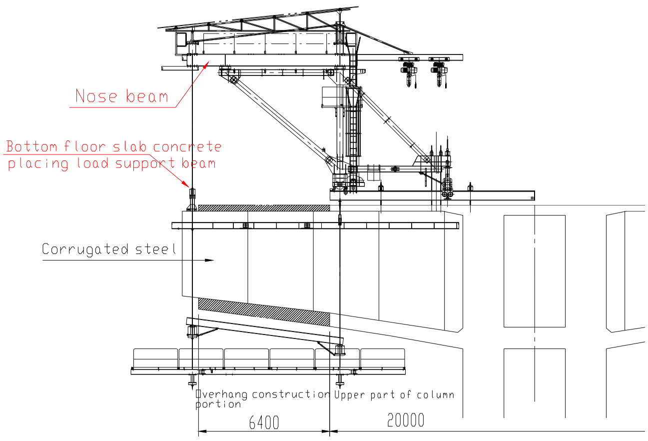


Figure 13. Mobile construction vehicle

Photo 5. Support beam for bottom floor slab concrete placing

6. CONCLUSION

The bridge was constructed as the second stage line, and the structure type of the bridge was changed to a corrugated steel web PC bridge, which is different from the first stage line bridge, because of on-site constraints. Also, through the application of double-wall-type piers, the adjustment of horizontal reaction, and the like, existing arch abutments were effectively utilized. These contributed to a reduction of the costs for constructing the entire bridge. In addition, approximately 40 days of work period was shortened by rationalizing the construction of the overhang. It would be a great pleasure if this report could be of any help to future bridge construction.

In closing, my deepest appreciation goes to all the people who cooperated in the design and construction of the bridge.